Contents

Chapter 4 Seismic Design and Retrofit

4.0 Seismic Design and Retrofit 4-1
4.1 General 4-1
4.2 Seismic Design Process 4-2
4.3 WSDOT Practices and Criteria 4-3
4.4 Elastic Dynamic Analysis (EDA) 4-6
- 4.4.1 Analytical Methods 4-6
- 4.4.2 Structural System or Global Analyses 4-7
- 4.4.3 Stand-Alone or Local Analysis 4-8
- 4.4.4 Model Verification 4-9
- 4.4.5 Effective Cracked Moment of Inertia for Reinforced Concrete Columns 4-9

4.5 Base Isolation and Energy Dissipation Devices 4-10
4.6 Retrofit Guide 4-12
- 4.6.1 Vulnerability Study 4-12
- 4.6.2 Material Properties 4-14
- 4.6.3 Earthquake Restrainers 4-14
- 4.6.4 Shear Blocks and Catcher Beams 4-15
- 4.6.5 Post-tensioning Strengthening 4-16
- 4.6.6 Column jacketing 4-17
- 4.6.7 Steel Structures 4-21
- 4.6.8 Base Isolation and Energy Dissipation Devices 4-22

4.99 Bibliography 4-23
Appendix 4-A
- Figure 4-A-1 Earthquake Probability - Poisson Model 4-A-1
- Figure 4-A-2 Design Examples of Seismic Retrofits 4-A-2

4.0 Seismic Design and Retrofit

4.1 General

The purpose of this chapter is to provide designers with an outline of the seismic design process and to provide specific WSDOT practices and criteria.

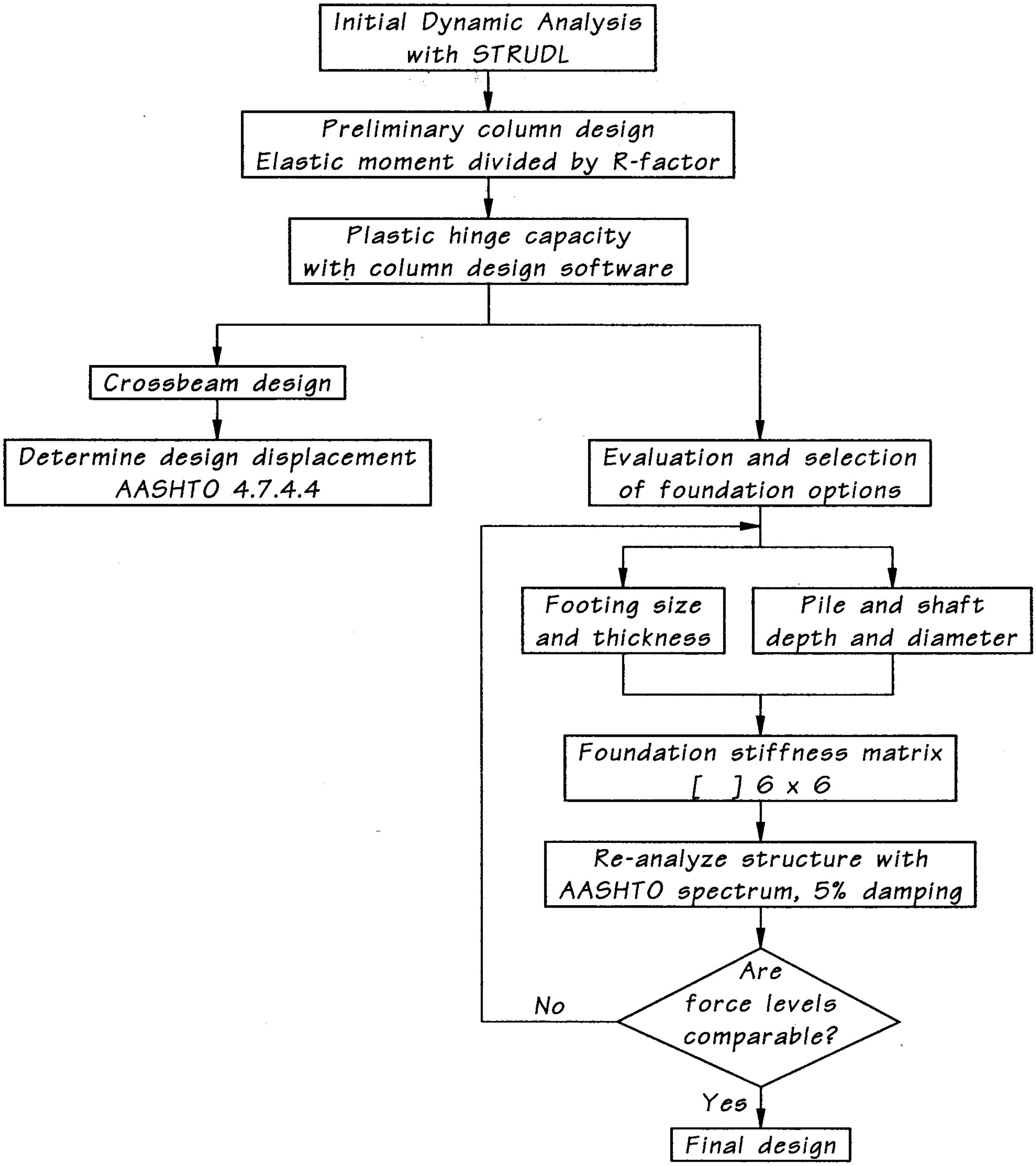
The format and material for this chapter is based on the *Seismic Design Criteria* (2004) developed by the California Department of Transportation (Caltrans). Additional references are: the National Highway Institute (NHI) Course No. 13063 *Seismic Bridge Design Applications* (1996), Publications FHWA-SA-97-006 through -012 (1996), and *Seismic Design and Retrofit of Bridges* (1996).

The latest AASHTO LRFD Bridge Design Specifications and all successive Interim Specifications are used for the seismic design of bridges. All highway bridges in Washington State are classified as “Others” except for special major bridges. Special major bridges fitting the classifications of either “Critical” or “Essential” will be so designated by either the Bridge and Structures Engineer or the Bridge Design Engineer.

MCEER (Multidisciplinary Center for Earthquake Engineering Research) has developed new seismic design criteria for bridge structures. These criteria involve two distinct levels: a Life Safety Level or less severe level with a 108-year return period and an Operational Level or more severe level based on a Maximum Credible Earthquake (MCE) with a 2,500-year return period. At this time, AASHTO has not adopted these new seismic criteria. See Appendix A for calculating the probability based on the return period. WSDOT has implemented the new MCEER seismic design criteria for only the design of specific large projects that involve substantial financial investment, such as the Alaskan Way Viaduct Replacement, using the Life Safety performance level only. It is expected that AASHTO will eventually adopt the new criteria. At that time, WSDOT will then use it for the design of new bridges.

4.2 Seismic Design Process

Figure 4-1 shows the seismic design process.



Seismic Design Process
Figure 4-1

4.3 WSDOT Practices and Criteria

Designers shall use the October 2002 USGS 10% Probability of Exceedance in 50 Year Seismic Hazard Contour Map (Figure 4-2) and the Zonation Map (Figure 4-3) in the seismic design of bridges.

The acceleration coefficients from these maps are used to determine seismic forces, which are applied in the horizontal direction only.

The current WSDOT design philosophy for dealing with seismically induced forces in bridge structures is to preserve life-safety through the prevention of collapse. The formation of plastic hinges in bridge columns, major changes in bridge geometry and even non-serviceability following the design level seismic event are all potential (and acceptable) results, but the structure must not collapse. WSDOT defines bridge collapse as follows:

> Bridge Collapse: Excessive movement or deformation of piers creating unacceptable bridge performance that may result in loss of life or serious injury. Unacceptable bridge performance includes, but is not limited to, partial or complete loss of span(s).

The WSDOT Geotechnical Design Manual, Section 6.1.1.1, further identifies the geologic hazards (liquefaction, lateral spread, downdrag, settlement) that may jeopardize highway bridges or structures during a design seismic event.

Earthquake restrainers shall be used across all expansion joints. The purpose of this is to tie the superstructure spans together, preventing the span from unseating and dropping due to insufficient existing seat length.

Designers shall use 5% damping on the AASHTO curves for the elastic spectral acceleration. The geotechnical engineer may develop special curves for the elastic spectral acceleration with higher damping if energy dissipation devices are installed on the bridge.

The passive resistance of the backfill soil behind the abutments shall not be included in seismic design.

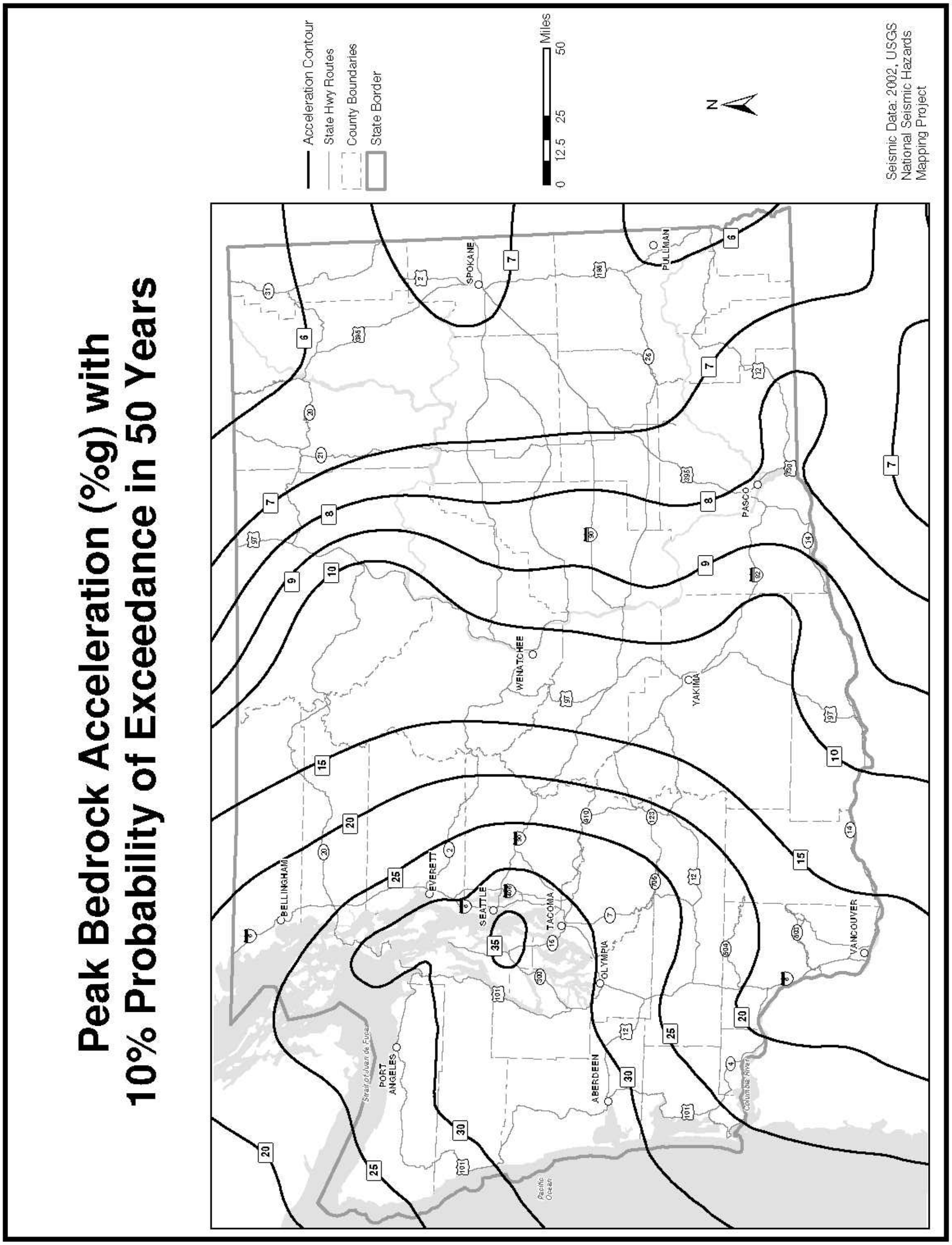
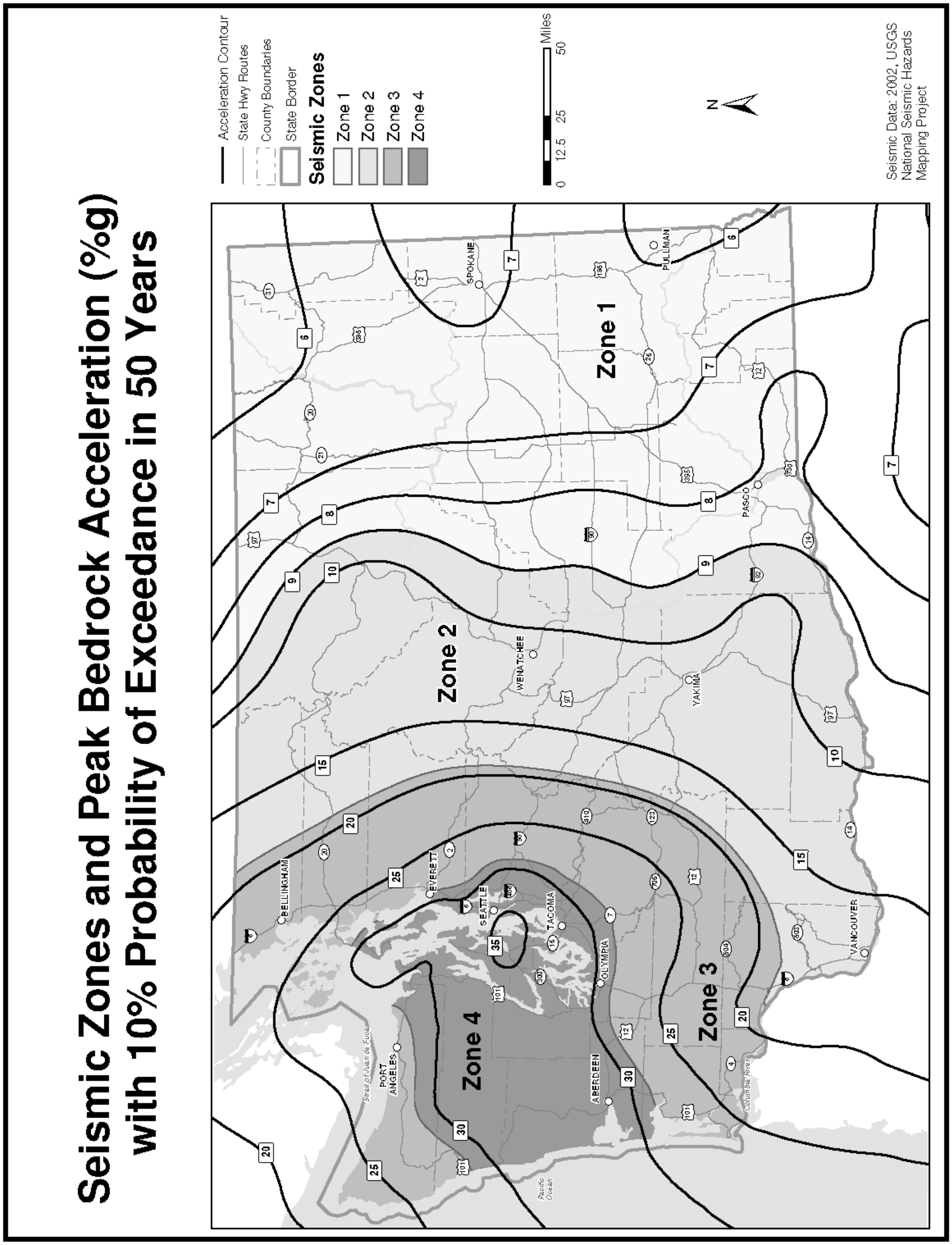

Peak Bedrock Acceleration (%g) with
10% Probability of Exceedance in 50 Years
Acceleration Contour
State Hwy Routes
County Boundaries
State Border
0 12.5 25 50 Miles
N
Seismic Data: 2002, USGS
National Seismic Hazards
Mapping Project
BELLINGHAM
EVERETT
SEATTLE
TACOMA
OLYMPIA
PORT ANGELES
ABERDEEN
VANCOUVER
WENATCHEE
YAKIMA
PASCO
SPOKANE
PULLMAN
Strait of Juan de Fuca
Pacific Ocean
Columbia River

Figure 4-2


Seismic Zones and Peak Bedrock Acceleration (%g)
with 10% Probability of Exceedance in 50 Years
Acceleration Contour
State Hwy Routes
County Boundaries
State Border
Seismic Zones
Zone 1
Zone 2
Zone 3
Zone 4
0 12.5 25 50 Miles
N
Seismic Data: 2002, USGS
National Seismic Hazards
Mapping Project
BELLINGHAM
EVERETT
SEATTLE
TACOMA
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PORT ANGELES
VANCOUVER
WENATCHEE
YAKIMA
SPOKANE
PULLMAN
PASCO
Strait of Juan de Fuca
Pacific Ocean
Columbia River

Figure 4-3

4.4 Elastic Dynamic Analysis (EDA)

4.4.1 Analytical Methods

The analysis methods and computer programs described in this section are based on a linear elastic multi-modal spectral analysis of the bridge and foundation as a structural system. This is defined as an Elastic Dynamic Analysis (EDA).

The EDA will likely produce stresses in some elements that exceed their elastic limit, indicating nonlinear behavior. For example, the stress-strain curve for concrete is nonlinear. Therefore, forces generated by a linear EDA could vary considerably from the actual force demands on the structure because of the non-linear behavior of some elements.

Other sources of nonlinear response are the effects of the surrounding soil, yielding of structural components, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior. Forces and moments shall be combined using the complete quadratic combination (CQC) method.

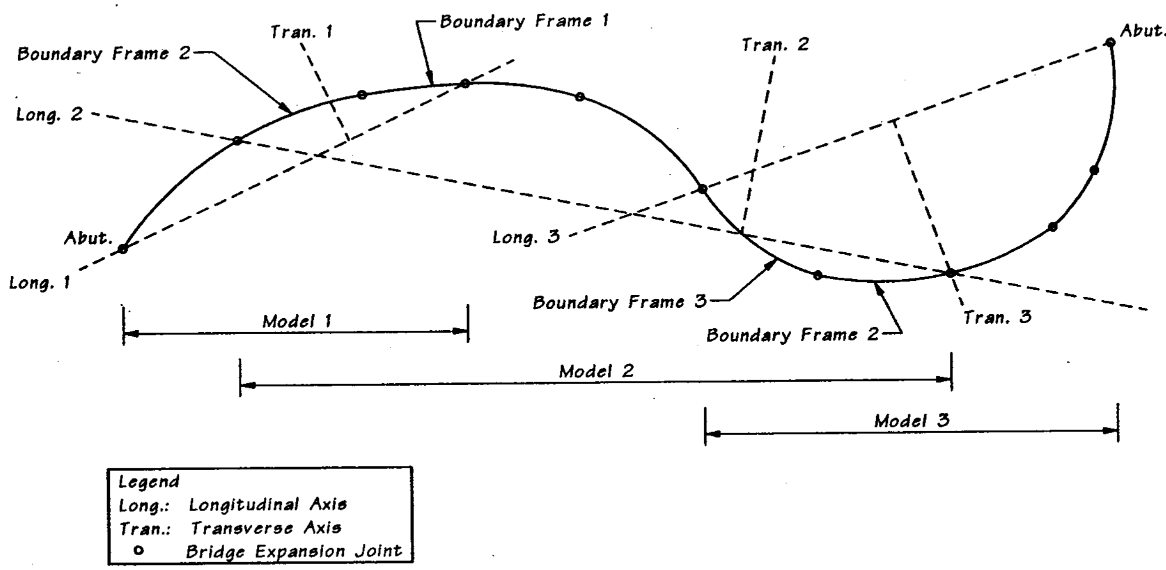
For straight multi-span bridges, the computer program SEISAB may be used to determine design forces. For more complicated structures, the computer programs GTSTRUDL or SAP2000 should be used. The number of degrees-of-freedom and the number of modes considered in the EDA should capture at least 90 percent of the mass participation in the longitudinal and transverse directions.

For bridges made up of many multi-span frames, the structure can be modeled by considering a portion of the structure. See Figure 4-5. However, a minimum of two boundary frames, or one frame and an abutment beyond the frame under consideration, shall be used for multi-span frame modeling.

Seismic Zone	Single Span Bridges	Multi Span Bridges	
		Other Bridges	
		Regular	Irregular
1	*	*	*
2	*	MM	MM
3	*	MM	MM
4	*	MM	MM

* No seismic Analysis required

Minimum Analysis Requirements For Seismic Effects
Figure 4-4



Frame Modeling Techniques
Figure 4-5

4.4.2 Structural System or Global Analyses

Structural system or global dynamic analyses are required when it is necessary to capture the response of the entire bridge system. Bridges with irregular geometry, curved bridges, skewed bridges, bridges with multiple transverse expansion joints, and massive foundation elements or foundations supported by soft soil, may exhibit dynamic response characteristics that are not necessarily obvious and may not be captured in a simple subsystem analysis.

Two separate global dynamic analyses are normally required to capture the assumed nonlinear response of a bridge because the bridge has two distinct dynamic behaviors: tension and compression.

In the tension model, the superstructure joints including the abutments are released longitudinally with truss or tension elements connecting the joints to model the effects of the restrainers. In the compression model, all of the truss or tension elements are inactivated and the superstructure elements are locked longitudinally to capture structural response modes when the joints close or when the abutment is mobilized.

The geometry of the bridge will dictate if both a tension and compression analysis is required. Bridges with appreciable superstructure curvature may require additional models, which combine the characteristics identified for the tension and compression models.

Long multi-frame bridges shall be analyzed with multiple elastic models. A single multi-frame model may not be realistic since it cannot account for out-of-phase movement between individual frames and may not have enough nodes to capture all of the significant dynamic modes.

Each multi-frame model should be limited to five frames plus a boundary frame or an abutment on each end of the model. Adjacent models shall overlap each other by at least one useable frame. See Figure 4-5 for details.

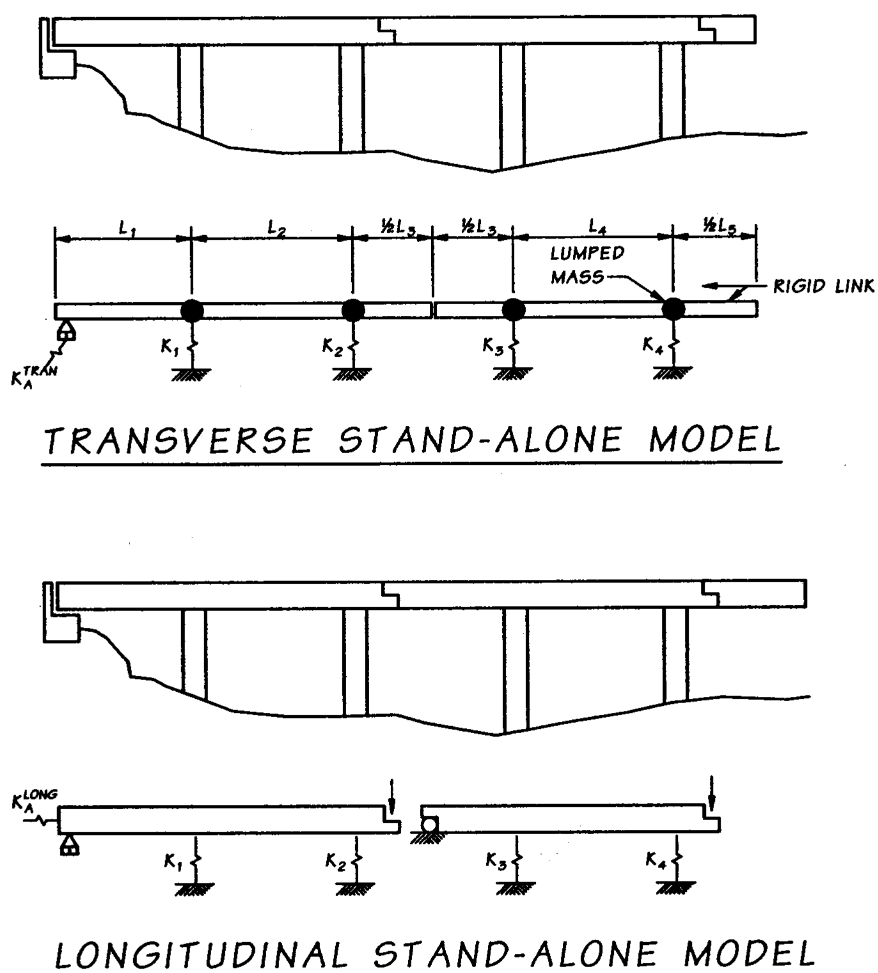
The boundary frames provide some continuity between adjacent models but are considered redundant and their analytical results are ignored. A massless spring should be attached to the dead end of the boundary frames to represent the stiffness of the remaining structure. Engineering judgment should be exercised when interpreting the deformation results among various sets of frames since the boundary frame method does not fully account for continuity of the structure.

4.4.3 Stand-Alone or Local Analysis

A stand-alone or local analysis is used to model the strength and ductility capacity of an individual frame, bent, or column. A stand-alone analysis shall be performed in both the transverse and longitudinal directions.

A transverse stand-alone frame model assumes lumped masses at the columns. The lumped mass shall include the mass of the upper half of the column and shall be located at the intersection of the superstructure center of gravity and the column centerline. Hinged spans shall be modeled as rigid elements with half of the superstructure mass lumped at the adjacent columns. See Figure 4-6. The transverse analysis of end frames shall include a realistic estimate of the abutment stiffness consistent with the abutment's expected performance. The transverse displacement demand at each bent in a frame shall include the effects of rigid body rotation around the frame's center of rigidity.

Longitudinal stand-alone frame model shall include the short side of hinges with a concentrated dead load, and the entire long side of hinges supported by rollers at their ends, see Figure 4-6. Typically the abutment stiffness is ignored in the stand-alone longitudinal model. This provides conservative forces at the intermediate piers.



Stand-Alone Analysis
Figure 4-6

4.4.4 Model Verification

The computer results shall be verified to ensure accuracy and correctness. The designer should use the following procedures for model verification:

- Using graphics, check the orientation of all nodes, members, supports, joint and member releases. Make sure that all the structural components and connections correctly model the actual structure.
- Check dead load reactions with hand calculations. The difference should be less than 5 percent.
- Calculate fundamental and subsequent modes by hand and compare results with computer results.
- Check the mode shapes and verify that structure movements are reasonable.
- Increase the number of modes to obtain 90 percent or more mass participation in each direction. SEISAB/GTSTRUDL/SAP2000 directly calculates the percentage of mass participation.
- Check the distribution of lateral forces. Are they consistent with column stiffness? Do small changes in stiffness of certain columns give predictable results?

4.4.5 Effective Cracked Moment of Inertia for Reinforced Concrete Columns

The effective cracked moment of inertia, I_{eff}, should be used when modeling reinforced concrete columns in the seismic analysis. I_{eff} shall be approximately 0.5 I_g, where I_g is the gross section moment of inertia of the concrete section.

More accurate I_{eff} may be determined from the empirical curves, depending on the axial load ratio and A_{st} / A_g ratio. See Figure 85, FHWA, *Seismic Retrofitting Manual for Highway Bridges*, pg. 200.

4.5 Base Isolation and Energy Dissipation Devices

A. Base Isolation

The decision to use base isolation devices must be approved by the Bridge Design Engineer.

For most concrete bridges with two to four spans, the period of vibration for the primary mode is approximately one second or shorter. This short period produces large seismic forces in the superstructure and substructure based on the AASHTO acceleration spectrum. Base isolation should be considered during the design phase if the seismic forces are too high and result in very large column dimensions, very large foundations or very deep foundations.

The use of base isolation may result in more economical foundations because base isolation lengthens the period which reduces the acceleration and lowers the seismic forces. This leads to designs with smaller members and less mass. Some types of base isolation devices dissipate seismic energy through internal friction or re-crystallization to further reduce the seismic forces.

Many types of proprietary isolation devices are available with various dynamic properties and characteristics. Each type may be applicable for only certain types of structures. The devices should be evaluated as to their effectiveness and design results obtained before being presented to the Bridge Design Engineer for approval of use. All devices are required to be tested under contract. Testing requirements shall be included in the Special Provisions.

The design criteria for base isolation devices should follow the AASHTO *Guide Specifications for Seismic Isolation Design* (1999).

B. Energy Dissipation Devices

Energy Dissipation Devices (EDD's) or dampers can shift the period of the structure and dissipate energy through additional damping. The AASHTO acceleration spectrum is based on 5% structural damping.

A higher viscous damping will lower the seismic forces to be resisted by the structure. An increase in the equivalent viscous damping reduces the spectral acceleration and displacements. See Figure 4.5.4a & b.

Damping ratios of 20% to 30% may be achieved using EDD's. The geotechnical engineer shall develop the acceleration spectrum for various potential damping ratios achieved by the EDD's used in the design.

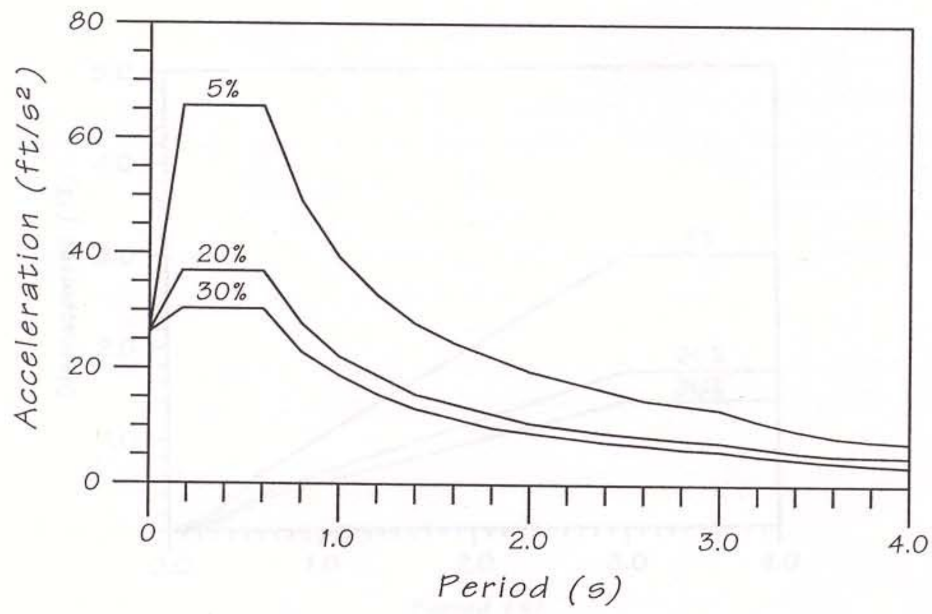
Base isolation devices and dampers may be combined in bridge design to reduce the seismic forces and displacement.

C. Shock Transmission Units (STU's)

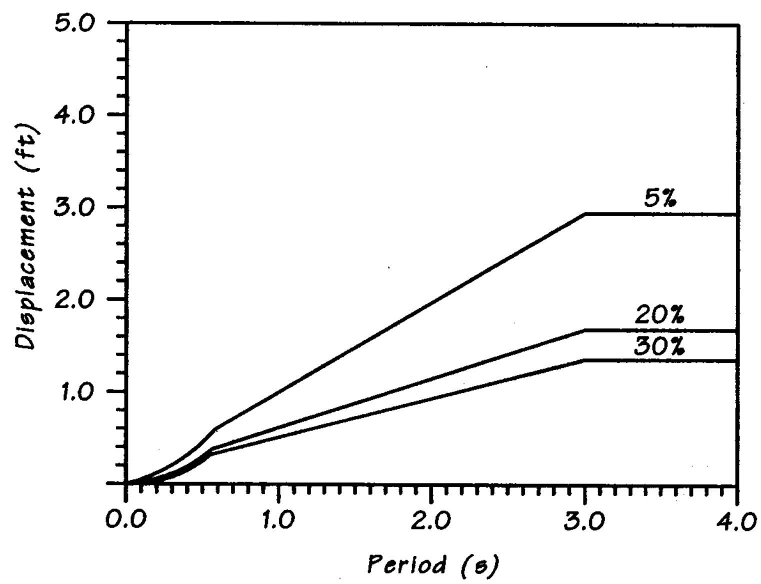
A shock transmission unit (STU) is a proprietary lock-up device which provides a compression and tension connection between the superstructure and substructure during an earthquake. STU's are frequently located at the tops of piers with existing sliding bearings. The STU's allow slow movement, such as thermal expansion and contraction without resistance, but lock up as a rigid compression or tension link to redistribute load when activated during a seismic event.

A structural analysis should be performed to determine the locations for the STU's and the forces in the STU and structural elements during an earthquake.

All STU's shall be tested under contract prior to installation and the testing requirements shall be included in the Special Provisions.



Acceleration response spectrum for medium-dense soil as a function
of viscous damping (ground acceleration = 0.8g)
Figure 4-7a



Displacement response spectrum for medium-dense soil
as a function of viscous damping (ground acceleration = 0.8g)
Figure 4-7b

4.6 Retrofit Guide

4.6.1 Vulnerability Study

A Seismic Vulnerability Study is required prior to beginning a seismic retrofit design project. For bridges in Seismic Categories B, C and D, the study shall identify collapse mechanisms and critical structural components that will be damaged by a design level earthquake (see Section 4.3). The seismic retrofit shall upgrade these structural components so they can resist an earthquake without structural collapse or loss of life.

Seismic retrofit alternatives shall be developed with input from the Bridge and Structures Office Seismic Specialist during the Seismic Vulnerability Study and shall be submitted to the Bridge Design Engineer for approval. The Bridge Design Engineer will consider approval of the seismic retrofit alternatives if a structure warrants a higher protection level because of its importance to the transportation system.

The Seismic Vulnerability Study begins with a complete review of the existing structural details and includes a structural analysis.

Some particular details of the structure, which should be reviewed, are:

- Expansion joints
- In-span hinges and AASHTO seat width requirements (See AASHTO Section 4.7.4.4)
- Adequacy of shear reinforcement, particularly size and spacing for short columns
- Confinement of column reinforcement to include anchorage into the concrete core
- Inadequate lap splices of column longitudinal reinforcement
- Welded end-to-end longitudinal reinforcement
- Transverse shear blocks at abutments or intermediate piers
- Narrow gaps between tall columns and adjacent structure

The purpose of the dynamic structural analysis is to determine the forces in the members. These forces and moments are the demand imposed on the members from the earthquake. Separately, the capacity of the members is calculated. For reinforced concrete columns, the computer program RECOL can be used to calculate the capacity. Next, the Demand/Capacity (D/C) ratio can be calculated. The decision on which members to seismically retrofit can be determined from the D/C ratios. All members with D/C greater than 1.0 are vulnerable and shall be strengthened so that the D/C is equal to or less than 1.0.

Earthquakes are multi-directional. Therefore, the designer should recognize that forces from the dynamic analysis could be reversed. To avoid incorrect results, the designer should watch for the reversal of signs of the seismic forces when superimposing these forces with member dead load.

Geotechnical information is provided by the Geotechnical Services Branch – Materials Laboratory and should contain information to calculate the foundation stiffness for dynamic structural analysis. L-pile Plus, the WSDOT “Design Manual for Foundation Stiffness under Seismic Loadings”, and “S-Shaft” are used to calculate the foundation stiffness.

Sometimes, new test borings are needed to obtain up-to-date soil information. The potential for differential settlement and lateral spreading caused by soil liquefaction will also be addressed in the Geotechnical Report and should be reviewed and evaluated to identify foundation vulnerability. Stone columns or other soil liquefaction measures may be occasionally considered, but will require the approval of the Bridge Design Engineer prior to actual use in the design. Retrofitting foundations is very expensive. At this time, the seismic retrofit of foundations is not included in the WSDOT Seismic Retrofit Program.

The designer should notify the Bridge Design Engineer if the foundation components have D/C ratios greater than 1.0. This may have a significant impact on the decision to proceed with the seismic retrofit.

A Linear Analysis

A linear analysis is used to obtain the forces and moments acting on structural components in order to calculate D/C ratios. Seismic retrofit of the bridge will not be necessary if both the flexural and shear D/C ratios of all the structural components of a bridge are equal to or less than 1.0. Seismic retrofit of the bridge is required when the shear or flexural D/C ratio of one or more structural components is larger than 1.0. The computer program GTSTRUDL is commonly used to perform a linear static analysis to determine the demand forces, moments, and deflections of a bridge.

The R-factors are not used to reduce the demand values for the D/C analysis. The demand values from the linear analysis should be used directly in the D/C calculations.

For reinforced concrete columns, the designer should use 50 percent of the gross column moment of inertia in the linear analysis to represent the cracked section of the column. See Section 4.4.5 if a more refined stiffness is warranted.

Soil-structure interaction shall be used in this analysis by including the foundation stiffness matrix.

B. Pushover Analysis

A pushover analysis is used to study the sequence of the formation of plastic hinges and the global instability of a bridge. When one or more plastic hinges form, the seismic forces will be reduced and re-distributed resulting in fewer columns requiring retrofitting. If no plastic hinges result from the pushover analysis, then retrofit of the bridge may not be necessary or the number of columns that require retrofitting may be reduced.

A pushover analysis is a nonlinear static analysis to study the failure mechanism of a structure. Pushover analysis may be done by iterative hand calculations or by computer software. The latest version of GTStrudl and SAP2000 have the capability to perform pushover analyses. Pushover analyses provide a reasonable measure of the nonlinear earthquake behavior of frame-type structures without having to perform a nonlinear time-history dynamic analysis.

The lumped plasticity approach, which assumes that yielding or plastic hinges form only at the ends of a member, is used in pushover analyses. A force-moment interaction yield surface provides the plastic hinge element. Bridge columns designed and built in the 1960's or 1970's with poor structural confining details should be input as having "no confining reinforcement" in the pushover analysis.

The structure shall be "pushed" by force control or displacement control. The structure is pushed in a given pattern until a predefined displacement or force magnitude is reached. When the strength of a structural element degrades to the extent it can no longer support its gravity load, local collapse occurs. When sufficient hinges are formed causing the structure to become unstable, global collapse occurs. Sometimes, the analysis may terminate if a global collapse mechanism develops before the prescribed loading or displacement is reached.

The displacement demand applied during the pushover analysis in the longitudinal direction will be the maximum longitudinal displacement determined from the load cases multiplied by 1.5. For a review of the load cases, see AASHTO Section 3.10.8 Combination of Seismic Force Effects. Simultaneous transverse displacement is neglected. Thus, the pushover analysis is a two dimensional analysis and is conducted separately at each bent in either the longitudinal or transverse direction.

C. Nonlinear Time History Dynamic Analysis

The Nonlinear Time History Dynamic Analysis should only be used to study the vulnerability of major bridges, such as: suspension bridges, cable-stayed bridges, and long-span truss bridges.

At least three records of time history input files, representing deep earthquakes, shallow earthquakes, and long duration subduction earthquakes, should be used in the nonlinear analysis to capture the worst-case scenario.

A Nonlinear Time History Dynamic Analysis is a nonlinear analysis using a recorded earthquake, which is divided into many small time increments. The seismic responses in terms of forces and displacements will be output at each of the time steps. The maximum forces and displacement of the structural components are summarized at the end of the analysis.

4.6.2 Material Properties

A. Concrete

The concrete strength specified for the original design does not reflect the current strength because concrete continues to gain strength over time. A 50 percent increase in concrete compressive strength should be used to calculate the modulus of elasticity and to evaluate the shear and flexural capacity of the column.

For example, a 4500 psi compressive strength may be used to calculate the capacity of the structural component if the concrete compressive strength used in the original design was 3000 psi.

Concrete experiences cracking and deterioration caused by alkali-silica reaction, salt-water intrusion, freeze-thaw and other environmental forces. For concrete bridges experiencing such deterioration, it may be appropriate to take core samples to determine the compressive strength of the cores in order to establish the actual in-situ concrete strength and the cross section losses, if any, of the members.

B. Reinforcing Steel

The mild steel reinforcement used in the 1960's and 1970's was Grade 40 with a yield strength of 40 ksi. A 44 ksi yield strength should be used for member strength analysis and capacity calculations because the actual tested yield strength is assumed to be higher by 10 percent.

4.6.3 Earthquake Restrainers

Span unseating is a common problem for existing bridges when the seat width is not sufficient. As a retroft measure, longitudinal earthquake restrainers are used to tie bridge superstructure sections together at in-span hinges and at locations with expansion joints.

For the design of new bridges located in Zones 2, 3 or 4, longitudinal earthquake restrainers shall be used across all expansion joints.

Longitudinal restrainers are high strength bars with both ends anchored on both sides of the adjacent units. A minimum two-inch gap should be maintained at one end of the restrainer to allow for thermal movement. High strength cable may be utilized if the rod cannot fit because of complex geometry, such as: a curved bridge or the movable portion of a ferry terminal.

Bridge Special provision BSP022604.GB6 specifies the current material requirements for the high strength steel bars.

Transverse restrainers are provided to prevent shear failure of the longitudinal restrainers during an earthquake. If the longitudinal restrainers cross a concrete or steel diaphragm, the holes in the diaphragm should be at least one inch larger than the diameter of the high strength steel bars. The transverse restrainer shall limit the bridge transverse movement to less than 1/2 inch.

Earthquake restrainers shall be designed in accordance with the Caltrans method and checked with AASHTO LRFD Section 3.10.9.5. See the earthquake restrainer design example in the Appendix B.

4.6.4 Shear Blocks and Catcher Beams

A. Shear Blocks

Transverse shear blocks shall be installed at all bearings to resist the seismic lateral forces in the transverse direction.

Occasionally, steel pipes filled with grout are installed at the intermediate piers and the in-span hinges of box girder bridges. The pipes should cross the two adjacent diaphragms, and one end of the pipe should be anchored at one of the diaphragms. A two foot square access hatch at the bottom of the box shall be provided to allow for the installation and inspection of the pipes.

Mild reinforcement dowels are drilled and set into the existing concrete with epoxy bonding agent. The potential sliding surface should be roughened to provide additional shear resistance. A shear strength reduction factor of ϕ equal to 1.0 shall be used when designing shear blocks.

B. Catcher Beams

The purpose of installing catcher beams is to provide additional seat support to prevent loss of the span due to span unseating. Most of the highway bridges built in the 1960’s and 1970’s do not have adequate seat widths. Many of these bridges have roller bearings, which together with insufficient seat width may cause the span to “roll” off the seat and drop.

Catcher beams are subjected to large forces during an earthquake due to the superstructure dropping and sliding on the catch beam. The catcher beams should be designed to resist the following two loading cases: (1) a vertical load of twice the dead load reaction plus the maximum live load reaction, and (2) a vertical load equal to the dead load reaction together with a horizontal load equal to the dead load reaction times the acceleration coefficient.

Concrete catcher beams shall be designed to transmit loads directly to the foundation if possible. Catcher beams anchored to an existing vertical face of concrete with dowels or anchor bolts are not considered to be reliable because of the large vertical and horizontal forces to which they will be subjected in the event the superstructure falls off its bearing onto the catcher beam. Consideration should be given to post-tensioning the catcher beams when direct load transfer to the foundation is not feasible.

The design of concrete catcher beam should provide for access for inspection and drainage requirements.

Steel wide-flange beams may be used at L-abutments if the required seat extension is not more than two feet. Special attention is needed for attaching the steel catcher beams to the abutment to provide additional support for the overhanging portion of the catcher beam.

All the catcher beams should have a ½-inch gap between the bottom of the superstructure and the top of the catcher beam. Steel shim stacks shall be used to adjust the cross slope and vertical curve for steel catcher beam. Web stiffeners should not be used. Instead, the designer should increase the size of the steel section if necessary to gain a thicker web.

4.6.5 Post-tensioning Strengthening

External post-tensioning is another way to increase the flexural capacity of concrete beams, girders and columns. Post-tensioning can also be used for concrete seat extensions and to provide column confinement for large flat surfaces.

A. Superstructure

Post-tensioning retrofits of reinforced concrete girders or crossbeams shall be designed for the moment demand of the structural component. Post-tensioning tendons shall be installed in the new concrete on both sides of the existing member. The same number of tendons should be used on each side to avoid eccentrically applied moments. Sufficient reinforcing bar dowels shall be provided to ensure adequate shear transfer between the existing and new concrete sections. Post-tensioning ducts shall be filled with grout after the post-tensioning.

Post-retrofit analysis shall be performed to check load re-distributions and structural adequacy on the columns and the adjacent superstructure elements.

B. Columns

Post-tensioning may be used as an alternative to steel jacketing in some special cases. Post-tensioning tendons shall be anchored into the crossbeam or pier cap and the footing.

Structural components that are retrofitted with post-tensioning will perform elastically during earthquakes. Plastic hinges shall not be allowed because stresses beyond yielding may cause collapse of the structure.

The flexural capacity after the retrofit is:

$$\sigma_{flexural} = (P_{axial} + P_{post\text{-}tension})/A \pm M/S$$

where: $P_{axial} = P_{DL} \pm P_{EQ}$

The shear capacity after retrofit is:

$$V_{total} = V_c + V_s + 0.2\,(P_{axial} + P_{post\text{-}tension})$$

C. Concrete Seat Extension

Post-tensioning is an alternative for bearing seat extensions when direct load transfer to the foundation is not feasible.

The bearing load on the seat extension shall be resisted by shear resistance of the concrete, shear reinforcing bars, and if needed, post-tensioned high strength steel bars in core drilled holes. New concrete shall be placed before the high strength steel bars are post-tensioned. The jacking force of the high strength steel bars shall be no more than 90 percent of its yield stress. Holes shall be filled with epoxy bonding agent after post-tensioning.

The total shear capacity of the seat extension is:

$$V_{total} = V_c + V_s + 0.2\,P_{post\text{-}tension}$$

The surface of the existing crossbeam or pier cap should be roughened to increase shear friction between the existing beam and the new concrete extension.

D. Confining a Flat Face of a Column

Steel jackets with large flat surfaces may be used only if circular or oval shaped jackets are not feasible.

High-strength steel bars should be installed perpendicular to the flat surface and anchored on both sides of the column because the flat surface of the steel jacket cannot provide the required "hoop" stress confinement of a circular or oval shaped steel jacket. A stress level of 25 to 50 percent of the yield stress in the high strength steel bars should be sufficient for the retrofit.

4.6.6 Column jacketing

A. General

It has been observed after past earthquakes, as well as during experimental tests, that columns are vulnerable to brittle failure, which could be prevented by adding steel jackets to the outside of the columns.

Column jacketing permits ductile behavior of the bridge during an earthquake by providing additional shear capacity and confinement and allowing for the formation of plastic hinges at one or both ends of the column. Forces and moments from a linear structural analysis shall be used to design the column jackets. See design example in Appendix B.

There are several deficiencies in columns constructed before 1973, which make them vulnerable to brittle failure. Two of these are: a) inadequate length of lap splices of longitudinal reinforcement where laps of $20d_b$ to $35d_b$ were typically used; b) inadequate shear reinforcement and confining reinforcement which consisted of single hoops overlapped 6 to 9 inches at the ends. Not all the structures constructed prior to 1973 have seismically vulnerable details. Designers must review the "as-built" plans to identify the column vulnerabilities and retrofit needs.

The type of connections at both ends of the column must be identified before designing a column retrofit. The superstructure-to-pier connection could be a moment-resisting connection, or a one-directional hinge, or one- or two-directional bearing. The substructure-to-column connection could be moment-resisting connection or one-directional hinge. Column retrofits may not be necessary if shear capacity of the columns are sufficient for columns having moment-resistant connections.

The decision to retrofit columns is based on the Demand/Capacity (D/C) ratio. If the flexural or shear D/C ratio of a column is larger than 1.0, the column should be retrofitted. If the D/C ratio is less than 1.0, the column does not have to be retrofitted.

The column should be retrofitted if the column has inadequate lap-splices of longitudinal reinforcement at one or both ends. The minimum requirement for lap-splices is given in Equation (6-3) of FHWA *Seismic Retrofitting Manual for Highway Bridges* (1995).

B. Plastic Hinge

The plastic hinge region will experience large deformation during earthquake resulting in concrete cracking, spalling and longitudinal reinforcement yielding. Therefore, openings and drilling of holes into and through the steel column jackets are not allowed in plastic hinge regions.

The analytical plastic hinge length is the equivalent length of column over which the plastic curvature is assumed constant for determining the plastic rotation.

$$L_p = 0.08\,L + 0.15\,f_{ye}\,d_{bl} \geq 0.3\,f_{ye}\,d_{bl} \qquad \text{(English units: inches, ksi)}$$

where:

L_p = distance from critical section from the plastic hinge to the point of contraflexure after development of a plastic mechanism

L = distance from transverse or longitudinal seismic force in the superstructure to the plastic hinge at the bottom of the column

d_{bl} = diameter of longitudinal reinforcement

f_{ye} = design yield strength for longitudinal reinforcement in plastic hinge

The plastic hinge region, L_{pr} defines the portion of the column that requires additional lateral confinement. L_{pr} is defined by the larger of:

- 1.5 times the cross sectional dimension in the direction of bending
- The region of column where the moment exceeds 75% of the maximum plastic moment, $M_{p\ col}$

C. Aesthetics Issues and Environmental Concerns

The Principal Architect in the Bridge and Structures Office should be consulted regarding the aesthetic considerations and selection of column retrofit types to match both the existing structure and the environment, especially for retrofit projects in urban areas.

For bridges on the list of federal or state historical records, the designer should discuss the retrofit design ideas with the Region Design Project Engineer’s Office and determine if coordination with the State Historical Preservation Office (SHPO) is required before beginning the final design.

Column retrofits in water or wetlands need to be coordinated with the HQ Bridge Construction Engineer, the Region Design Project Engineer’s Office, and Region Environmental Office because of the long lead-time to acquire permits. The impact area, construction methods, and duration should be determined and sent to the Region Design Project Engineer’s Office and Region Environmental Office when the Region begins work on the Design File.

If the construction of column retrofit has impact on a navigation channel, the US Coast Guard Liaison Engineer in the Bridge and Structures Office should be contacted for advice on how to proceed.

D. Column Retrofit Methods

1. Steel Jacketing

Steel jacketing has proven to be the most cost effective method for retrofitting circular and rectangular reinforced concrete columns in Washington State. The steel jacketing method should be the first choice unless it is not feasible.

Circular, oval and “D” shaped steel jackets may be utilized depending on the structural features and the shape of the cross section of the existing columns. Thick plates or high strength steel bars may be used on the large flat face of the “D” shape jacket to provide adequate confining pressure. Columns with variable cross section along its height or with flairs at the top should be analyzed to study the retrofit needs before designing the steel jacket. The steel jacket with either the variable cross section or uniform cross section steel jacket could be used in those cases in order to gain the most economical solution, if approved by the Principal Architect in the Bridge and Structures Office.

Partial height jacketing may be used to retrofit very tall columns or columns with hinge connections on the top.

The steel jacket should be designed according to FHWA *Seismic Retrofitting Manual for Highway Bridges* (1995) and CALTRANs *Seismic Design References* (1997).

The thickness of the steel jacket is calculated to meet the most severe requirements based on shear force, lap splices of longitudinal reinforcement and confining pressure in plastic hinge zone. The critical case for jacket design is when the lap-splices lose anchorage at the 0.001 radial strain level. The minimum thickness for a steel jacket is ¼-inch.

Typically, a space of two inches is provided at the bottom of the jacket and the top of the footing. A two-inch minimum and four inch maximum gap is provided at the top of the jacket and the crossbeam. The purpose of these gaps is to prevent the jacket from acting in compression and bearing against the supporting member at large drift angles.

Grout is pumped into the gap between the steel jacket and the existing column. The gap between a circular column and the steel jacket is ¾-inch minimum and 1-inch maximum. For square or rectangular columns, the ¾-inch minimum and 1-inch maximum gap should be maintained at the four corners. Spacers shall be installed to ensure the gap prior to pumping grout.

Pumping height should be limited to 20 feet for circular column and 8 feet for square or rectangular or split columns. The pumping holes should be located at the bottom of each of the pumping sections. Multiple pumping holes at each of the pumping sections should be installed for circular columns 6 feet in diameter or larger, or for square or rectangular columns in order to have evenly pumped heights of grout.

The grout, mortar, and epoxy bonding agent used for column seismic retrofit shall be as specified in the Bridge Special Provision BSP022606.GB6.

2. Fiber Reinforced Plastic (FRP) Wrapping

   FRP wrapping may be used for column retrofits in special cases to ease constructability problems. FRP has a high tensile strength and is lightweight. The design concept of FRP wrapping is the same as the steel jacketing. The required thickness of the composite materials depends on the fiber content and the strength of the fiber.

   Glass fiber wrapping is not suitable for column retrofit in Western Washington because of its strength reduction in high humidity or immersion application.

   Carbon FRP wrapping is a good alternative to a steel jacket for circular columns and square or rectangular columns with the maximum dimension of the cross section less than three feet. FRP wrapping should not be used if it will be abraded or rubbed.

3. Pedestals

   Most of the pedestals in Washington State are made of plain unreinforced concrete. Pedestal retrofit is required when: $W - D \leq 1'\text{-}0''$, where W is the width of the pedestal and D is the diameter of the column. If the width of the pedestal is larger, it will not require retrofitting.

   A shallow steel jacket is the most cost effective method for pedestal retrofit. The steel jacket for a pedestal retrofit should be designed using the same column retrofit concepts.

The steel jacket should be installed directly on the top of the footing without any vertical gap, and be 2 inches higher than the top horizontal surface of the pedestal. The horizontal gap between the jacket and the pedestal should be filled with grout. The 2 inches vertical gap should be maintained between the top of the pedestal and the bottom of the column jacket.

4. Split Columns

   Split columns were used on highway bridges in Washington State in the 1950's and 1960's to allow for thermal and shrinkage movements in long bridges made up of multiple frames. Generally, there are two split columns supported on a common footing with a 2-inch gap between the two split columns for thermal movement. Split columns may be split for the full length from the bottom of the superstructure to the top of footing or partially split at the top of the column and then joined together monolithically as a single large size column for the lower portion.

   According to the research report *Retrofit of Split Bridge Columns* (2000), full height split columns may be retrofitted with two "D"-shaped steel jackets. For a partial height split column, the column shall be retrofitted with two "D"-shaped steel jackets on the split portion and a circular or oval shape steel jacket on the non-split portion. The research report recommends that a six-inch vertical gap be maintained between the bottom of the two "D"-shaped steel jackets and the top of the circular or oval shaped steel jacket for the retrofit of a partial height split column.

5. Underground "Silo"

   A "Silo" should be used when the bottom of the steel jacket is buried more than 6 feet under the existing or final ground line, in order to leave room for column lateral deflection during an earthquake. PVC pipes or rigid polystyrene strips or panels may be used depending on the depth of the soil and the lateral deflection.

   If a steel jacket is installed in the concrete or asphalt pavement and the buried depth is less than 6 feet, premolded joint filler shall be installed between the jacket and the pavement.

   A "Silo" may also be used to change the overall structural performance during an earthquake by more evenly distributing the seismic forces between all the columns of a bridge.

6. Excavation Limit

   Horizontal excavation limits should be 4 feet measured from: a) the face of the circular column; b) the corner of the square or rectangular column; c) the face of the circular pedestal; or d) the corner of the square or rectangular pedestal, depending on the scope of the retrofit work. These excavation limts shall be shown in the plans.

   The need for excavation beyond these limts should be reviewed during the design phase of the PS&E development.

   Deep excavations are very expensive. If the required excavation is 18 feet or deeper, the designers should meet with the geotechnical engineers in order to develop a more practical solution.

7. Drainage Pipe and Utilities

   All of the existing drains and utilities (and all other attachments) that may have to be relocated, removed or reattached in the seismic retrofit project shall be shown in the Plans. Structural details should be provided if the drains and utilities require modifications to existing structural components. Strengthening the opening area with a welded steel doubler plate will be required where drainage pipes exist inside the column and exit through the steel jacket. Openings in and through the steel jacket within the plastic hinge zone are not allowed (See BDM Section 4.6.6B.

4.6.7 Steel Structures

Steel bridges in California and Washington States were severely damaged during recent earthquakes and are just as susceptible to damage from earthquakes as concrete bridges.

A SeismicVulnerability Study is necessary to determine the retrofit needs and method for steel bridges. Particular attention should be paid to the connections, bearings, and structural stability.

The seismic retrofit of steel bridges may include istallation of longitudinal seismic restrainers, and retrofit of in-fill walls and pier walls, bearings, steel superstructure members, and movable bridges.

A. Longitudinal Seismic Restrainers

Longitudinal seismic restrainers shall be installed at all piers with expansion joints, at in-span hinges, and at drop-in span locations. The cross section of the restrainer should be determined in the Seismic Vulnerability Study.

The earthquake restrainers should be anchored to the bottom side of the concrete deck or the main member of the steel structure by bolts or welded anchor plates. The locations of the anchors should be carefully studied to ensure the effectiveness and constructability; particularly, for curved bridges, or bridges with large skews, or bridges with other complicated geometry.

B. In-Fill Walls and Pier Walls

In the past, some steel bridges were built with two exterior columns connected by an in-fill wall or full-length pier walls to minimize snagging of debris.

From past earthquakes and lab tests, these pier walls may be damaged by lateral shear or pull out of the longitudinal pier wall reinforcement from the footing if the lap splices of the longitudinal reinforcement are not sufficient.

It is difficult to provide the required confinement by using steel jackets to form plastic hinge because of the long length of the in-fill wall or pier wall. The seismic retrofit of these pier walls should be by strengthening to keep the stress of the pier wall in the elastic range.

The most effective retrofit strategy is to thicken the in-fill wall, which will provide additional shear resistance, and to ensure elastic behavior.

C. Bearings

Steel rocker bearings were commonly used in the 1960’s and earlier for steel bridges. Shear resistance is provided by steel pintles, which could be sheared off causing the span to translate longitudinally and possibly drop. These rocker bearings should be replaced and retrofitted by installing catcher beams or transverse shear stops around the bearings. If elastomeric bearings are used as replacement bearings, the designer shall verify that the elastomeric bearing can accommodate the live load rotation of the superstructure.

Concrete cracking and spalling around the anchor bolts of rocker bearings has been observed following recent moderate earthquakes. In a strong earthquake, the bearings could lose their hold-down capability and lateral shear resistance resulting in the span dropping. FRP wrapping or steel collars should be installed to protect the anchorages of vulnerable bearings. The design of the FRP or steel collar should be similar to the steel jacket for columns.

D. Steel Superstructure Members

Diagonal members of trusses, cross frames, and secondary bottom lateral bracing members have buckled in past earthquakes. If the earthquake was stronger, the structure could be severely damaged.

All steel superstructure members should be studied to identify those with potential buckling failures. All vulnerable members should be replaced or retrofitted to prevent buckling.

E. Movable Bridges

Movable bridges, including lift bridges, have suffered severe damage and have encountered operational difficulties after moderate to strong earthquakes. Some observed damage to structural components included: railings, towers for lift spans, central locking mechanism, support frame for counterweight for bascule bridges, counterweights jumping the tracks for lift span bridges, and displacement or settlement for swing-span bridges.

The effective retrofit for these movable bridges is usually strengthening or replacement. The impact of any proposed retrofit on weight changes and on the mechanical/electrical system should be carefully studied. The proposed retrofit should be reviewed and approved by the Bridge Preservation Office's Mechanical and Electrical Engineers.

4.6.8 Base Isolation and Energy Dissipation Devices

A. Base Isolation

Column jacketing is an efficient and economical way to retrofit substructures. However, some bridges were built on wetlands or areas containing hazardous materials. It will be very expensive to excavate the hazardous materials and dispose of it. The construction access in wetlands has significant impact on the environment and will be very expensive to mitigate. It is very time consuming to obtain environmental permits. In these situations, base isolation may be a viable and economical retrofit option.

A feasibility study and analysis are required to determine the type and size of the isolation devices. Dampers and STU's may also be used in conjunction with base isolation if an analysis shows some of the structural components are still vulnerable when only isolation devices are used.

Isolation devices should be able to resist traffic braking and wind loads without excessive deflection. These isolation devices must also allow thermal expansion to occur in the superstructure without overstressing the substructure, and shall be stable during an earthquake. The devices should maintenance-free.

Larger safety factors should be used in isolation design. All the devices should be tested under contract prior to installation. The design criteria for base isolation devices should follow the AASHTO *Guide Specifications for Seismic Isolation Design* (1999).

Liquefiable soil layers underneath footings or pile foundations act as a natural base isolation device because the shear waves cannot transmit through the liquefied soil layer. A geotechnical engineer will have to develop a special acceleration spectrum based on the liquefiable soil layer.

B. Energy Dissipation Devices and STU's

Energy Dissipation Devices (EDD's) and STU's can be used to reduce the seismic forces and displacement. A structural analysis should be performed to determine the location and sizes required for dampers and STU devices. All dampers and STU devices should be tested under contract prior to installation

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